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Featuring

Design and Construction of the Florianopolis Bridge

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Volume XIII NOVEMBER, 1926 Number 9

THE DESIGN AND CONSTRUCTION OF THE FLORIANOPOLIS BRIDGE

By D. B. Steinman and W. G. Grove

(Presented October 20, 1926, at a Joint Meeting of the Boston Society of Civil Engineers; M. I. T. Student Chapter, American Society of Civil Engineers; Harvard Engineering Society; Tufts Civil Engineering Society; and the Northeastern University Section, Boston Society of Civil Engineers.)

D. B. Steinman*

The Florianopolis Bridge, with a main span of 1,113′ 9″, is the longest span bridge in South America and the longest eyebar suspension span in the world. It is of especial interest in being the first application of a new form of suspension stiffening construction. It is also of interest in being the first structure in the world to utilize a recently developed structural material, — high-tension, heat-treated steel. Another novel feature is the use of rocker towers, — towers designed to rock at the base. Attention is also called to the ingenious and novel method of erection which was developed for this structure.

History of the Project

To my mind, however, far more compelling than the technical interest of these various novel and distinguishing features is *the human interest of the story of the Florianopolis Bridge;* for the history of the project, from 1920 to 1926, is a story of six years of high adventure and romance in the planning and building of a bridge, — a dramatic story of vision, hopes, disappointments, opportunities seized, and consummation realized. The bridge was the ambitious life-dream of a Governor in a remote province in Brazil. After overcoming all obstacles and finally launching the project, he suffered the tragedy of dying before he could see the fulfillment of his vision. His last hours were made happier by his view of a smaller replica of the bridge which his people erected in the park facing his windows. From that far-away Brazilian province, the action of the story extends to other countries in both Hemispheres, where engineers and construction companies were making competitive designs and estimates, and were pinning their hopes on the success of their plans. It extends to Wall Street, where the fall of a large banking house meant the death of the project until new millions were raised to replace the funds that had been lost. It extends to engineering offices in New York, where faith in a bridge type was rewarded with alternations of disappointment and success. It returns to Florianopolis, where American ingenuity wrestled with problems and overcame difficulties that threatened the success of the work. Such were the far-reaching ramifications — the far-flung scenes — in the drama of this bridge project. From the American bridgemen, who married Brazilian girls, to the Brazilian contractors, whose expected fortune in profits evaporated in a manner not uncommon in South American political affairs, there are many in whose lives the Florianopolis Bridge has played a dramatic rôle.

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* Consulting Engineer, 25 Church Street, New York.

The bridge was built for the State of Santa Catharina, a province near the southernmost corner of Brazil. This Brazilian state has the peculiar circumstance of having its capital city, Florianopolis, located on an island in the Atlantic Ocean. The island suffered from lack of physical connection to the mainland. The bridge was wanted to provide highway connection, railway connection and water supply from the mainland to the island city of Florianopolis.

Even prior to 1920 engineers and contractors in various parts of the world were working on designs and estimates for this bridge. A. Y. Sundstrom, a Brazilian contractor who had received his engineering education and training in the United States, finally succeeded in securing for his firm the general contract from the local government. He had made a preliminary cantilever design, which he sent to the American Bridge Company to develop and estimate. He also wrote to Holton D. Robinson, an authority on suspension bridges, concerning the possibility of a suspension design being more economical. Dr. Robinson called on me and invited me to join forces with him in preparing a suspension design that would compete with the cantilever. (That call, incidentally, was the commencement of our acquaintance which later ripened into our present professional partnership.) We had faith in the suspension type for the design of the Florianopolis Bridge, and proceeded to make preliminary studies. Shortly thereafter Mr. Sundstrom arrived in New York, and, upon seeing our studies, authorized us to complete our suspension design.

In the meantime the American Bridge Company engineers completed their cantilever design, and both designs were submitted to that Company for bidding. To our intense disappointment, the suspension bid was a little higher than the bid on the cantilever design, and the latter was adopted.

The necessary capital had been raised through a bond issue, and everything was in readiness for proceeding with the project, when, suddenly, the New York banking house that held the bulk of the funds went into bankruptcy. With the vanishing of these resources, the project died.

ADOPTION OF THE SUSPENSION TYPE

Two years later (in 1922) new funds were raised and the project came to life again. Mr. Sundstrom once more came to New York. Minor revisions were made in our suspension design and in the cantilever design of the American Bridge Company, and prices were obtained from several steel companies. This time, in gratifying vindication of our faith, the suspension design won out by a substantial margin, and was adopted. In fact, all concerned had, by this time, reached the conclusion that the suspension type was the economical design for the Florianopolis Bridge. Thus were hopes and disappointments reversed.

Mr. Sundstrom's firm, holding the general contract, contracted with the U. S. Steel Products Company to furnish the steelwork, and proceeded to secure prices, in this country and abroad, on the wire for the cables. The lowest figure was received from a wire firm in England. Mr. Sundstrom was just about to cable acceptance of this bid when the American Bridge Company (through the U. S. Steel Products Company) asked for another opportunity. The suggestion, originating with Mr. Wm. G. Grove of the American Bridge Company, was that high-tension heat-treated steel eyebars be substituted for the wire cables. This was a new material that had been developed through several years of experimental work by the American Bridge Company, and had never before been used in any structure. The guaranteed minimum ultimate strength was 105,000 lbs. per sq. in., the guaranteed minimum elastic limit was 75,000 lbs. per sq. in., and the recommended working stress was 50,000 lbs. per sq. in. The Steel Corporation offered to furnish the eyebars of this material and to erect the cable at a price that would equate the total cost of the structure to our lowest figure for the total cost using wire cables. This proposition looked attractive to our client, Mr. Sundstrom, particularly since it relieved him of the cable-erection contract. We pointed out, however, that the substitution of the heavier eyebar construction would increase the quantities and cost of the towers and anchorages on account of the increased dead load to be supported; whereupon the Steel Corporation agreed to make a deduction from their figure to balance these items. Upon satisfactory completion of these price adjustments, the subcontract was awarded to the U. S. Steel Products Company to furnish and erect these eyebars. Following this, in order to avoid duplication of erection organization and equipment, the entire superstructure erection was added to this contract.

When this decision was made — to substitute heat-treated eyebars for the wire cables — the thought occurred to us to take advantage of the eyebar construction by introducing a modified form of stiffening construction that would be more efficient than the previous conventional design. A new form was developed and adopted, yielding four times the rigidity of the previous design with only two-thirds as much steel in the stiffening truss. The revision offered a considerable saving in cost throughout the structure, and it promptly won the approval of our client. The Steel Corporation accepted the revision in design, — although it meant reduced tonnage and revision of erection plans, — and the new design was adopted.

Thus it is seen how the official design of the Florianopolis Bridge passed through successive changes, alternating between the cantilever and suspension types, and winding up as a complete revision of the suspension design which had won the competition. The revision made even greater the economic advantage which had determined the selection of the suspension design over the cantilever.

Recently we received a letter from a distinguished engineer in Germany, congratulating us on the Florianopolis design. In this letter he informed us that he was one of the competitors in the early days of the project; he stated, however, with good international sportsmanship, that he was glad that our design was adopted, since it was so much more interesting than the suspension design which he had prepared.

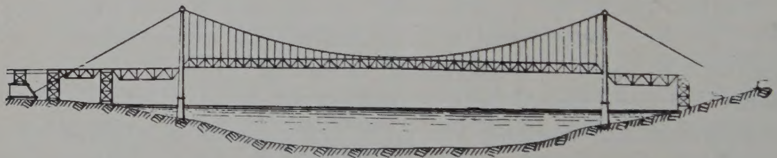
DEVELOPMENT OF NOVEL SUSPENSION DESIGN

In developing the novel suspension design which was adopted for the Florianopolis Bridge, the following were the governing considerations:

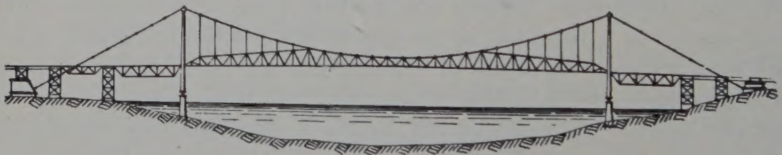
In the central portion of the conventional form of suspension construction, the cable, which sustains a heavy tensile stress, is in close proximity to the upper chord of the stiffening truss, where compression is the governing stress. Such juxtaposition of two principal elements carrying opposing stress represents a waste of material, or, rather, a neglected opportunity for increasing economy. By combining the two opposing structural elements a great saving can be effected, — the result is a subtraction of stresses instead of an addition of sections. There is thus secured a partial neutralization of the maximum tensions in the middle portion of the cable, and the corresponding portion of the upper chord of the truss is dispensed with.

Another neglected opportunity for increasing economy and efficiency in the conventional form of suspension construction is represented by the use of parallel-chord stiffening trusses. For maximum economy the truss should have a profile conforming to the variation of maximum bending moments along the span, a principle which receives recognition in the design of other structures, such as simple trusses, cantilevers, continuous trusses and arches. Since the economic depth at any section is a function of the governing bending moment, a truss should have its greatest depth at the points of greatest bending moment, and should be made shallow where the bending moments are comparatively small. In a suspension-bridge stiffening truss, the greatest bending moments occur near the quarter-points of the span; consequently the economic profile of a stiffening truss is one having maximum depth near the quarter-points and minimum depth at mid-span and at the ends. This conclusion is strengthened by the fact that the shears in a stiffening truss are a minimum near the quarter-points, and attain maximum values at the middle and ends of the span, so that a truss-profile with maximum depth near the quarter-points also gives economy in web members, since it provides the shallowest depth in the regions where the web stresses are greatest. Such a profile yields the additional advantage of greater uniformity of required chord-sections throughout the span. A wide range of variation in required sections generally involves a waste of material in those chord members where minimum sections are required.

Another consideration governing truss-depth is that of efficiency in reducing deflections. The most serious deflection of a stiffening truss, as measured by the resulting deflection gradients, is produced under the condition of live-load covering approximately one-half of the span. Half-span loading produces a downward deflection of the loaded segment and a somewhat smaller upward deflection of the unloaded segment, with a maximum deflection gradient at the loaded end. The magnitude of this deformation depends upon the truss-depth at and near the quarter-points. Calculations show that to limit the deflection gradient to 1 per cent a truss-depth of 1/45 to 1/40 of the span is required. A parallel-chord stiffening truss of such depth (as illustrated by the Williamsburg Bridge) would render the structure unsightly. In order to secure the requisite stiffness without resorting to a stiffening truss of clumsy proportions, we must abandon the parallel-chord type and adopt an outline providing the extreme depth only where it is needed, namely, in the vicinity of the quarter-points of the span.



Original Design



Adopted Design

FIG. 1.— ORIGINAL (PARALLEL-CHORD) AND ADOPTED (CURVED-CHORD) DESIGNS FOR FLORIANOPOLIS BRIDGE

Adopted design secures great increase of rigidity with less material than required for parallel-chord design

The foregoing considerations show that the logical directions for improvement of conventional suspension design for increased economy and efficiency are along two lines: (1) utilization of the cable as the top chord of the stiffening truss (preferably limited to the middle half of the span); and (2) variation of truss-profile to give maximum depth near the quarter-points of the span. The two requirements, fortunately, are congruous; complying with the first automatically facilitates compliance with the second. The result is the new form of suspension construction adopted for the Florianopolis Bridge.

Comparative cost-estimates of the two designs shown in Fig. 1 demonstrate a material saving in favor of the new design. In addition to the major elements of economy outlined in the preceding general discussion, there are a number of incidental savings arising from the change in design. The new form of construction yields the following contributions to a reduction in the total cost of the structure:

(1) Saving the material represented by the middle half of the top chord of each stiffening truss.

(2) A general saving in the remaining chord material resulting from the use of an economic truss-profile conforming to the variation of bending moments along the span, and, in particular, a material reduction in the maximum chord-sections previously required in the vicinity of the quarter-points of the span.

(3) A saving in details and in minimum sections resulting from the greater uniformity of required chord-sections throughout the span.

(4) A saving in web material on account of the reduced truss-depth in the regions of maximum shear.

(5) Dispensing, in the middle half of the span, with the subverticals previously required to half-length the compression-chord members which are now replaced by tension members.

(6) Dispensing with the intermediate top laterals previously required to half-length the same compression-chord members in a horizontal plane.

(7) As a result of these various savings, a reduction of about one-third in the total weight of the stiffening truss, and a consequent saving in all portions of the structure affected by the dead-load of the truss.

(8) A saving in chain (or cable) sections resulting from the reduced dead-load of truss and the consequent reduced dead-load of chain.

(9) Saving of the suspenders in the middle half of the span and a reduction in length of the remaining suspenders.

(10) A reduction (6.5 ft. in the case of the Florianopolis Bridge) in the total height of the towers in consequence of the reduced distance between cable and lower chord.

(11) A saving in the towers resulting from the reduction in height combined with the reduction in dead-load of truss and chain.

(12) A material saving in the anchorages resulting from the reduced dead-load of truss and chain and the reduced elevation of the backstays.

In the case of the Florianopolis Bridge, a large portion of the economy yielded by the change of design was not capitalized, but was turned back into the structure in the form of reduced unit stresses to provide a greater margin of safety for future load-increase. Thus the design stress for the stiffening truss was reduced from 20,000 to 18,500 lbs. per sq. in. (actually 14,000 as calculated by the exact method);

and the unit stress in the chain was reduced from 50,000 lbs. per sq. in. to 46,500.

While the governing consideration in the change of design was that of economy, the change yielded greatly increased rigidity as an incidental advantage. The parallel-chord stiffening truss was 25 feet deep throughout; the revised layout, utilizing part of the cable (or chain) as top chord, provides a truss with depth varying from 22.5 ft. (at center and ends) to 42.5 ft. (at the quarter-points). The maximum calculated deflection under full-span loading is only 1.41 ft., or 1/800 of the span; and the maximum deflection under the critical half-span loading is only 1.02 ft., or 1/1100 of the span, with uplift in the unloaded half practically eliminated. These deflections are approximately one-fourth of the corresponding values for the previous (conventional) design. Since the Florianopolis Bridge is designed to carry a railway as its principal element of live-load, this reduction of the governing deflections is of practical significance.

About 25 per cent increase in rigidity may be attributed to the substitution of eyebars for wire cables; the remainder of the 300 per cent increase is the direct consequence of the new form of stiffening construction.

The following elements of the new design contribute to this increase in rigidity: (1) the revised truss-profile is more efficient in resisting deflections, since it provides maximum depth in the regions of greatest bending moment; (2) the depth at the quarter-points has been made nearly twice as great as in the previous design, and the stiffness in the vicinity of the quarter-points is the principal factor in determining the rigidity of a suspension bridge under the critical condition of half-span loading; (3) the functioning of the full section of the cable as top chord of the stiffening truss in the middle half of the span greatly increases the moment of inertia in that portion of the span; (4) the fact that live-load introduces tension in the middle half of the top chord (by virtue of its forming part of the cable) further reduces mid-span deflections.

As a result of these various factors, the change of design yields greater stiffness with less material in the structure. In approximate figures, the design is four times as rigid with only two-thirds as much material in the stiffening truss. Thus, greater efficiency has been secured through a more scientific design of the suspension stiffening system.

In addition to the marked increase in vertical rigidity yielded by the new design, the lateral stiffness is improved by the large cable (or

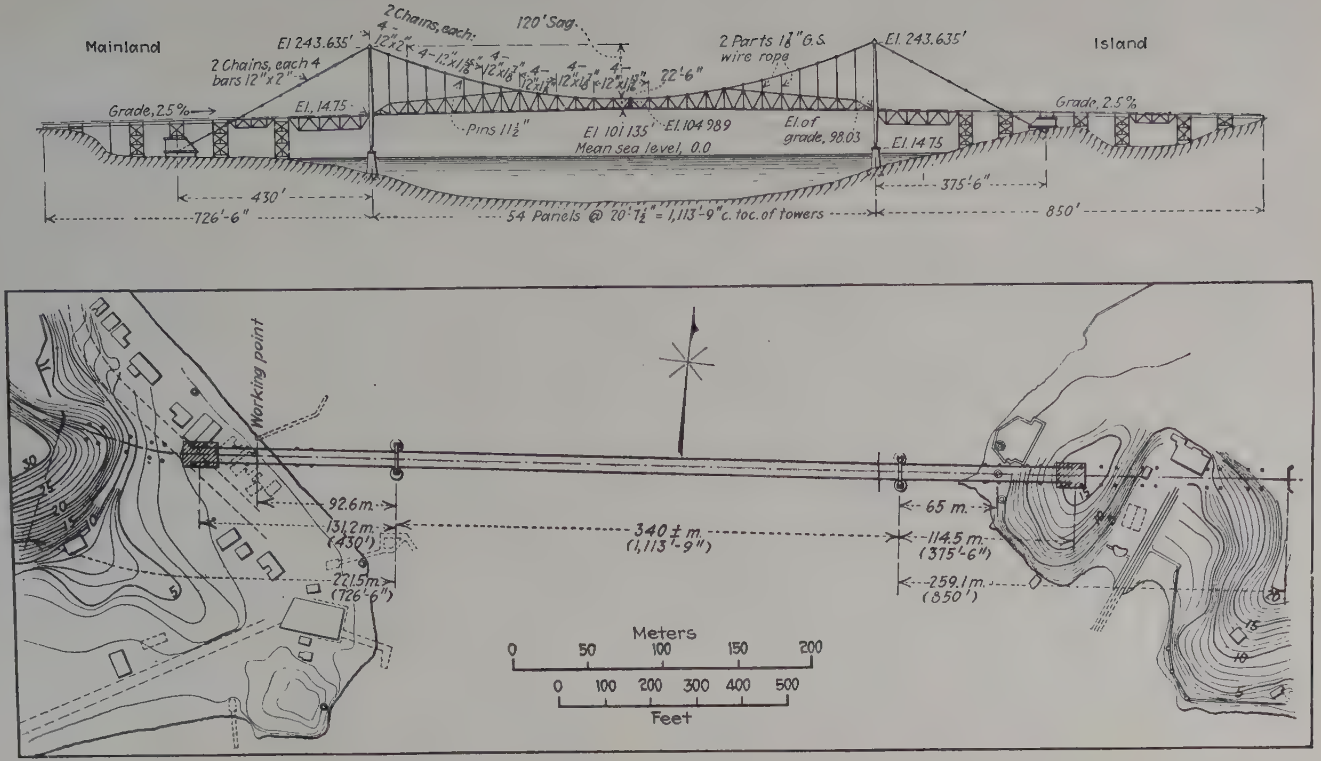


Fig. 2. — General Plan and Profile of Florianopolis Bridge

chain) sections functioning as wind-chords in the middle half of the span; and ideal longitudinal rigidity is secured by the direct connection of the truss to the cables. Longitudinal or braking forces are carried directly into the cable without producing bending moments in the towers.

The net result of the change in design as applied to the Florianopolis Bridge is a reduction in cost (through actual saving in material), an increase in safety and longevity (through lower unit stresses), and an increase in efficiency (as measured by resistance to deflections).

In reference to the recently completed Philadelphia–Camden Bridge, the chief engineer of that project has stated that the calculated deflection of 15 feet is not objectionable since it is less than 1/100 of the span. In the Florianopolis Bridge we have reduced the corresponding deflection to 1/800 of the span, not by increasing the quantity of metal, nor by using a more cumbersome construction, but by using less metal more scientifically disposed.

In the recent world-wide competition for the Sydney Harbor Bridge in Australia, the suspension type was excluded by Act of Parliament because the local engineer declared that it would be too flexible; he stated that, under the proposed loading of four main-line railway tracks, a suspension bridge of 1,600 ft. span would have a deflection of 11 feet. Accepting this challenge, our firm (Robinson and Steinman) was retained by Australian bidders to prepare a suspension design. We made it an extension of the Florianopolis type, and succeeded in reducing the maximum calculated deflection, under full-span loading, to 1.28 ft., or only 1/1250 of the span; under the critical half-span loading, the maximum calculated deflection was only 0.83 ft., or about 1/2000 of the span. These calculated deflections were checked by the Australian government's engineer, and he declared that our design was the most rigid suspension bridge that had ever been designed.

These comparisons serve to emphasize a principle that needs to be more generally known, — *suspension bridges can be designed to have any desired degree of rigidity.*

Use of Heat-treated Carbon-steel Eyebars

In the Florianopolis Bridge a new material, in the form of high-tension, heat-treated carbon-steel eyebars, finds its first application. This material, intended to be used with a working stress of 50,000 lbs. per sq. in., has been developed through recent experimental research by the American Bridge Company. It is furnished under guarantee of minimum elastic limit of 75,000 and minimum ultimate strength of 105,000 lbs. per sq. in., and minimum elongation of 5 per cent in 18 ft.

Since the previous experimental work had been confined to eyebars of 12 in. x 2 in. section, the eyebars for Florianopolis were made to conform to this section as nearly as possible, ranging from 12 by 1 13/16 to 12 by 2.

A novel detail was adopted for the Florianopolis eyebars in the form of oval pinholes. The hole is made somewhat elongated (axially) and enlarged in back to facilitate entry of the pin, while retaining close fit in the segment of contact. The front or bearing semicircle exceeds the diameter of the pin (11½ in.) by only 5/1000 in.; the rear semicircle is bored to the diameter of the pin plus 1/32 in., and the two centers are separated 1/8 in. along the axis of the eyebar. The unusually close fit thus secured along the bearing surface reduces the secondary stresses in the eyebar head. This detail is copyrighted by the American Bridge Company.

The 11½-in. pins for connecting the 12-in. eyebars are of special heat-treated steel, with a yield point ranging between 60,000 and 65,000 lbs. per sq. in. and a tensile strength ranging between 100,000 and 105,000 lbs. per sq. in. A few pins are of chrome-nickel steel, of same strength values.

Rocker Type of Tower

The Florianopolis Bridge will be the first American suspension bridge built with rocker towers. The only large bridges previously built with this feature are the Elizabeth Bridge at Budapest (1903), and the bridge over the Rhine at Cologne (1915).

The evolution of suspension-bridge tower design has been marked by three successive types. First came the *rigid* form of tower, typified by the masonry towers of the Brooklyn Bridge and earlier suspension bridges. The principle of rigid tower construction persisted for some time after steel replaced masonry, as illustrated in the towers of the Williamsburg Bridge. When engineers realized that the rollers on the tower tops were generally inoperative and could not be relied upon to provide the necessary movement of the cable saddles, the *flexible* type of tower was introduced. In that type, the saddles are fixed on the tops of the towers, and the horizontal movement necessitated by unbalanced cable pull is provided by flexure of the slender, fixed-base towers. First used in the Manhattan Bridge, the flexible type of tower has been applied in the more recent suspension structures, such as the Rondout, Bear Mountain and Philadelphia bridges. As a further step

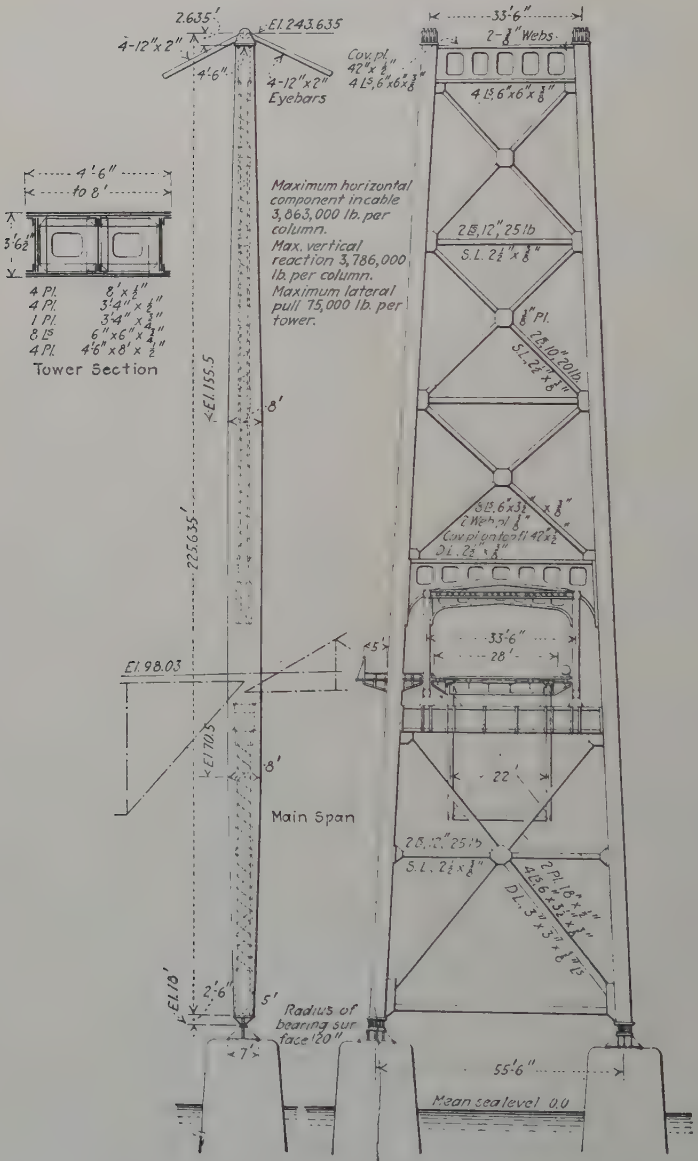


FIG. 3. — DETAILS OF ROCKER-TYPE TOWER

toward the reduction and simplification of tower stresses, the *rocker* type of tower has been developed.

The rocker type offers the most economical and scientific design for suspension-bridge towers. It eliminates the bending stresses from unbalanced cable pull, thereby yielding a saving in tower material, and it obviates the difficulties of the necessary erection operation of pulling back the tops of the towers prior to stringing the cables or chains.

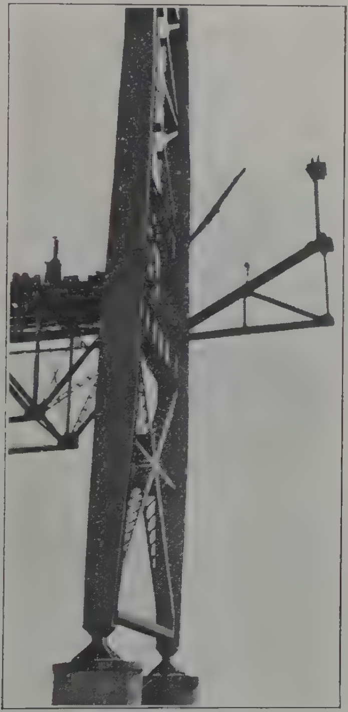


FIG. 4. — VIEW OF TOWER DURING ERECTION OF BRIDGE

In the case of the Florianopolis Bridge, the change was made from fixed base towers after comparative estimates showed a net saving of about 20 per cent in the cost of the towers in favor of the rocker type. There is a material reduction in the main sections by the elimination of the bending stresses.

Another important advantage is a material reduction in the required section of the piers, from the relief of bending moment which would otherwise have to be carried all the way down to the foundations.

A general drawing of the Florianopolis towers, showing the principal sections and dimensions, is presented in Fig. 3; and a photograph, during construction of the bridge, is shown in Fig. 4.

The towers are approximately 230 ft. high. The legs are battered, from a top width of 33′ 6″ to a width of 55′ 6″ at the base. The width at the top corresponds to the spacing center to center of trusses, the chains hanging in vertical planes. This form of tower design, with main legs battered, was introduced into suspension construction by Dr. Holton D. Robinson. Its chief advantage is the facility of running the truss and roadway construction through the tower portal without interference from the tower legs; an additional advantage is increased transverse stability, of particular importance in the case of narrow bridges.

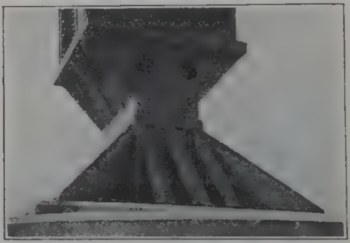


Fig. 5. — Rocker and Base Castings at Foot of Tower

The two legs of the Florianopolis tower design are braced together with rigid diagonal and transverse bracing members, the latter including a transverse distributing girder at the tower top, a transverse portal girder immediately above the stiffening truss portal, and a transverse reaction girder, supporting the stiffening truss and approach truss bearings.

Each tower leg or column is made up of a double-box section, having a maximum longitudinal width of 8 ft. and a constant transverse width of 3′ $6\frac{1}{2}$″. The 8-ft. width tapers to 4′ 6″ at the top, and to 5′ 0″ at the base. Transverse stiffening diaphragms are provided at intervals, two in each column section.

The tower column has a maximum cross-section of 370 sq. in., tapering to a minimum of 285.5 sq. in. at the top, and to 297.5 sq. in. at the base. This variation in cross-section (and in moment of inertia), similar to that provided in derrick-booms, is calculated to take care of the varying flexural stress produced by the long-column action.

The rocker and base castings are shown in Fig. 5. The pedestal or base casting, which rests on the concrete piers, is finished to a plane top surface 27 x 45 in. The rocker casting, which is affixed to the column base, is finished on its lower or bearing surface to a radius of 10 ft. The line of contact is 45 in. long. For security against any possibility of creeping displacement, four screw-dowels of 3-in. diameter are provided. The rocking of the upper casting on the lower was tested in the shop with the dowels temporarily in place. The bottom face of the pedestal casting is cast with two full-length diagonal lugs which engage corresponding grooves in the masonry to prevent any possible sliding of the casting.

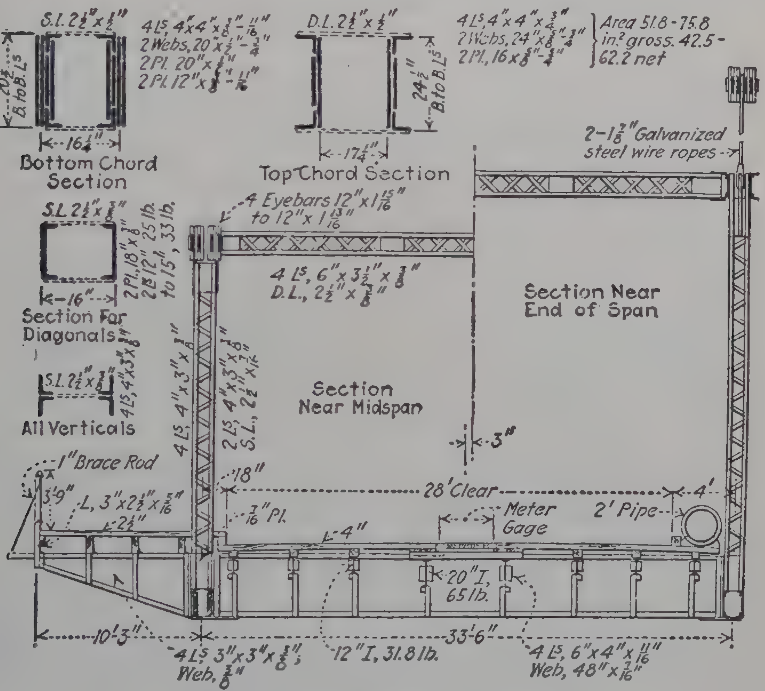


Fig. 6. — Cross Section of Bridge

The maximum vertical reaction on each rocker-bearing is 2,420 tons.

The bridge was required to carry a 28-ft. roadway, a meter-gage electric railway, a 24-in. water main, and a 9-ft. sidewalk. The adopted arrangement of cross-section is shown in Fig. 6. The cables and trusses are spaced 33.5 ft. center to center; the sidewalk is carried on an outside bracket along the north truss, and the water main is located just inside the south truss so as to help equalize loads.

On account of the inherent stability of the suspension construction, sway-bracing is dispensed with. Adequate systems of lateral bracing have been provided.

ANCHORAGES

The east anchorage, on the island of Florianopolis, is located on rock. The anchor chains and reaction girders are embedded in concrete in two stepped trenches excavated in the solid rock. The sides of these trenches were given a negative batter to increase the vertical resistance, and on this construction is superimposed the buttressed concrete anchorage structure. The west anchorage is situated on lower ground on the mainland. The excavation did not reveal rock where anticipated, and a pile foundation had to be used; about 25 per cent of the piles (located under the forward portion of the anchorage) are battered in the direction of the resultant pressure. Both anchorages are U-form in plan, for maximum efficiency.

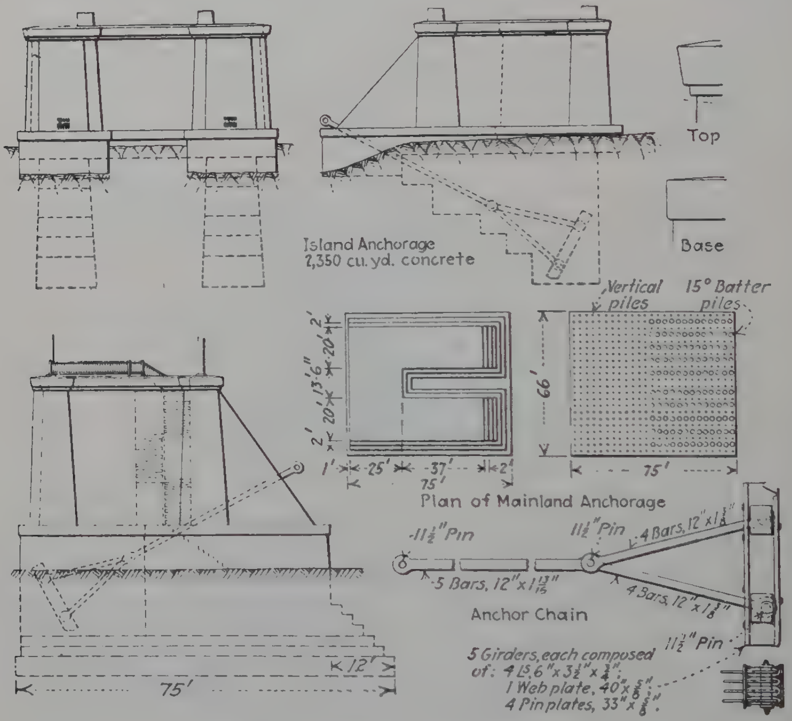


FIG. 7. — DETAILS OF ANCHORAGE

Each anchor chain, consisting of heat-treated eyebars, divides into two branches for connection to the anchor girders. Each anchor girder, 16 ft. long, consists of five built-up girders connected together by tie-plates and reinforced with pin-plates. At the Y-point of each anchor chain, a built-up pin-seat was provided to hold the pin in accurate position during the placing of the concrete around the anchor girder and the connecting eyebars. Above these points, the anchor chains were boxed in during the completion of the concreting in order to prevent adhesion of the concrete until full dead-load strain is in the chains.

The four main piers of the Florianopolis Bridge are cylindrical concrete shafts, 22 ft. in diameter, with simple coping. A separate pier cylinder is provided under each tower leg.

Construction was commenced in the spring of 1923.

Huge, irregular boulders in the ocean bed were found to make the rock soundings quite misleading. The presence of these boulders and the slope of the bedrock surface complicated the foundation work.

An open square cofferdam of steel sheetpiles was driven for each of the four main pier foundations. The sheetpiles, ordered to length based on the previous soundings, went down to a maximum of 38 ft. below mean water level, while rock lay at depths from 30 to 60 ft. below water.

A novel procedure was adopted in the case of the north continent pier. After excavation within the 28 ft. square cofferdam was carried down to near the bottom of the sheetpiles, a central test pit, 8′ x 8′, was sunk within the cofferdam to bedrock at Elevation —59. This test pit was concreted, and then adjoining sections were similarly excavated and concreted, in succession, until the entire area within the cofferdam had thus been underpinned down to bedrock. On the concrete footing so obtained, the square pier was built up, stepping off at different levels and terminating in the cylindrical shaft.

In the adjacent south continent pier cofferdam, rock was found at Elevation —32 over the greater part of the area, falling off sharply to Elevation —60 at one corner. Only this corner had to be sheeted down below the cofferdam sheeting.

The two island main piers reached rock at Elevations —37 and —50, respectively.

The entire substructure work, including the concrete piers and anchorages, was completed in June, 1924.

The Elizabeth Bridge, over the Danube at Budapest, with its span of 951 ft., previously held the record as the longest eyebar suspension span. It is an extremely interesting structure, structurally as well as architecturally. For its erection, cumbersome falsework was placed across the Danube, with heavy timber trusses spanning the openings that had to be left for navigation. The eyebars were supported on this heavy staging during their stringing. If a similar amount of falsework were required for every eyebar suspension bridge, there would be little prospect for the adoption of structures of this type. Fortunately, however, the American Bridge Company evolved an entirely different procedure for the erection of the Florianopolis Bridge. The eyebars were hung from temporary rope cables, and the stiffening truss members were also erected from these overhead ropes. All falsework was dispensed with; even the temporary footbridges ordinarily used in the construction of suspension-bridge cables were eliminated. It might be said with verisimilitude, that the structure was erected by means of "sky-hooks."

Summary of Design Features

To recapitulate: The Florianopolis Bridge is of interest —

1. In being the first application of a new form of stiffening construction, yielding greatly improved economy and efficiency.
2. In being the first structure utilizing the new high-tension, heat-treated steel, permitting a working stress three times as high as ordinary steel.
3. In being the first American suspension bridge employing rocker towers.
4. In being the first application of an improved method of erection employing no falsework or staging.
5. In being the longest eyebar suspension bridge in the world.

Comparative graphs also indicate that, in proportion to span and design load, the Florianopolis Bridge is the most efficient and economical suspension bridge ever built.

More than these technical distinctions, however, the human interest of the story of this adventure in bridge building may account for the widespread interest that has been shown in the Florianopolis Bridge.

William G. Grove *

Florianopolis, the capital city of the State of Santa Catharina, Brazil, is located on the island of Santa Catharina, which is separated from the mainland by a narrow strait. The island is about 25 miles long, and the city is located on the strait side of the island about midway from either end.

This strait is too shallow to permit the passage of large ocean-going vessels, although the coastwise Brazilian vessels can pass through. The city had no physical connection with the mainland, and there was no railroad connection on the mainland opposite the city.

To increase ocean trade with his native state one of the recent governors conceived a fourfold project:

1. Building a port at the northern end of the island.
2. Constructing about 12 miles of electric railway between the new port and the city.
3. Building a bridge connecting the island with the mainland.
4. Constructing about 100 miles of railroad on the mainland connecting the bridge with a railroad line running through the interior between Rio de Janeiro and Montevideo.

The general contract for the third part of this project, the construction of the bridge across the strait, was let by the State of Santa Catharina to the Brazilian firm of Byington and Sundstrom, and this firm asked the U. S. Steel Products Co. to design a bridge of the cantilever type.

While making the original design of the cantilever type of structure some of the American Bridge Company engineers became convinced that the suspension bridge was the more economical type of bridge for this location, and made independent studies on a suspension bridge design using high tensile heat-treated eyebars for the main cables instead of parallel wires or wire ropes. Recent experiments in heat treating of steel by C. W. Bryan, Chief Engineer, and C. G. E. Larsson, Assistant Chief Engineer of the American Bridge Company, had resulted in the production of a steel having an elastic limit of between 75,000 and 100,000 lbs. per sq. in. With this high elastic limit it is permissible to increase the unit working stress from 16,000 to 50,000 lbs. per sq. in., thereby requiring less than one-third as much steel as would be necessary in the use of the ordinary 30,000 lbs. per sq. in. elastic limit steel.

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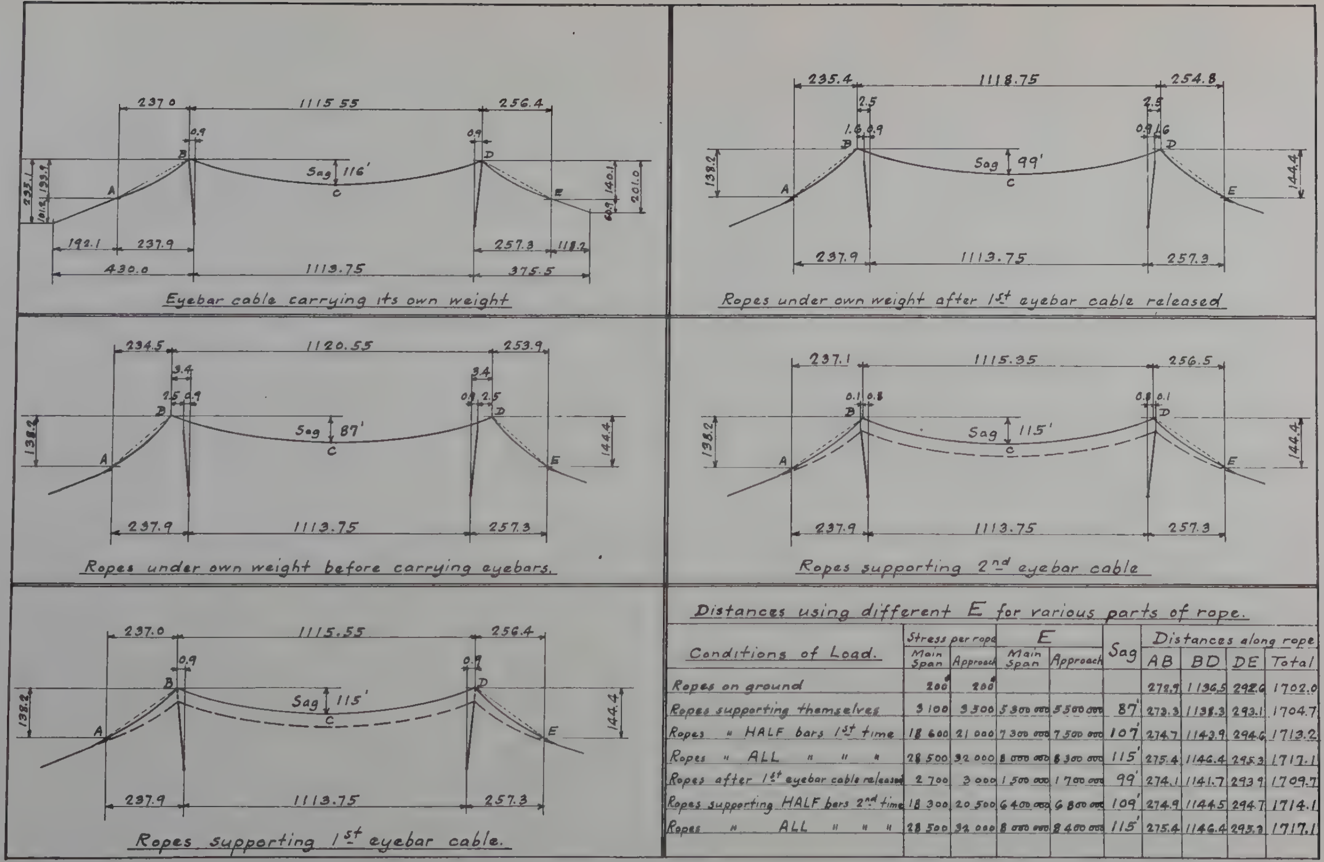
* Assistant Engineer, American Bridge Company, New York City.



Eyebar cable carrying its own weight

Ropes under own weight after 1st eyebar cable released

Ropes under own weight before carrying eyebars.

Ropes supporting 2nd eyebar cable

Ropes supporting 1st eyebar cable.

Distances using different E for various parts of rope.

Conditions of Load.	Stress per rope		E		Sag	Distances along rope			
	Main Span	Approach	Main Span	Approach		AB	BD	DE	Total
Ropes on ground	200#	200#				272.9	1136.5	292.6	1702.0
Ropes supporting themselves	3 100	3 500	5 300 000	5 500 000	87'	273.3	1138.3	293.1	1704.7
Ropes " HALF bars 1st time	18 600	21 000	7 300 000	7 500 000	107'	274.7	1143.9	294.6	1713.2
Ropes " ALL " " "	28 500	32 000	8 000 000	8 300 000	115'	275.4	1146.4	295.3	1717.1
Ropes after 1st eyebar cable released	2 700	3 000	1 500 000	1 700 000	99'	274.1	1141.7	293.9	1709.7
Ropes supporting HALF bars 2nd time	18 300	20 500	6 400 000	6 800 000	109'	274.9	1144.5	294.7	1714.1
Ropes " ALL " " "	28 500	32 000	8 000 000	8 400 000	115'	275.4	1146.4	295.3	1717.1

FIG. 8. — SAGS AND MODULI OF ELASTICITY OF ERECTION ROPES UNDER VARIOUS CONDITIONS OF LOADING

USE OF HIGH TENSILE HEAT–TREATED STEEL

As the high tensile heat-treated steel eyebars find their first application in the Florianopolis bridge, a brief description of this steel would be of interest. Heat treatment of steel has been known for some time but only recently has its application been made to steel bridges. The material used is carbon steel, but the per cent of carbon is somewhat higher than the carbon content in the 30,000 lbs. per sq. in. elastic limit grade commonly used for structural purposes. The steel is manufactured and rolled in the larger eyebar sizes ranging in width from 8 to 16 in. The eyebar heads are made in the usual manner by the eyebar machine at the Ambridge plant of the American Bridge Company. The bars are then placed in the heat-treating furnaces and subjected to temperatures necessary to produce elastic limits and ultimate strengths of the desired amount. They are then quenched, reheat-treated and allowed to cool slowly. Each bar is treated separately, the temperatures being taken by a series of pyrometers.

The first heat-treated bars placed on the market were of the 50,000 lbs. per sq. in. elastic limit grade. These bars are guaranteed to have a minimum elastic limit of 50,000 lbs. per sq. in., a minimum ultimate strength of 80,000 lbs. per sq. in. and a minimum elongation of 8 per cent in 18 ft. This grade of bars has been used during the past ten years in the Bessemer & Lake Erie Bridge, 1918; Cincinnati Bridge, 1922; Oil City Bridge, 1924; and in the double cantilever bridge with duplicate main spans of 1,100 ft., now being built (1926) over the Carquinez Strait, California, this latter bridge being designed by Dr. D. B. Steinman.

Since 1920, investigations were made toward raising the elastic limit and by the end of 1922 the success of the experiments was such that the American Bridge Company felt warranted in recommending for use in the Florianopolis Bridge, cables made of heat-treated eyebars guaranteed under full-size tests to have a minimum elastic limit of 75,000 lbs. per sq. in., a minimum ultimate strength of 105,000 lbs. per sq. in. and a minimum elongation of 5 per cent in 18 ft. It was this development of high tensile steel which had placed structural steel eyebars on a comparable basis with plain wire for the main cables.

The recommendation to use eyebars for the cables was accepted by the general contractors, and it was found that the cable stress was such as to require 4 bars 12 in. wide, varying in thickness from $1\frac{13}{16}$ in. at the center of the span to 2 in. at the towers.

The general scheme of erection was to stretch a temporary wire

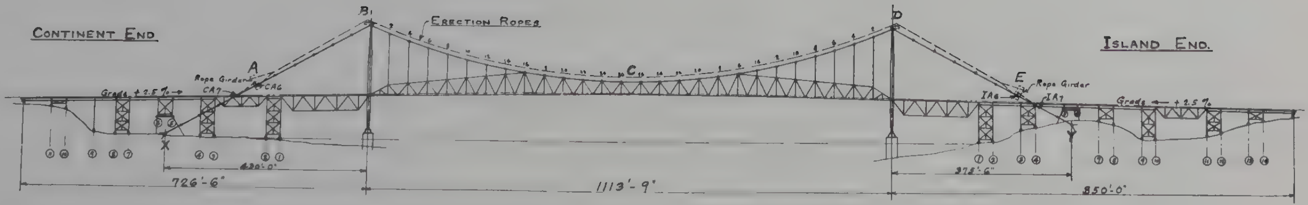


FIG. 9. — GENERAL PROFILE OF BRIDGE, SHOWING POSITION OF ERECTION ROPES, ROPE GIRDERS, SADDLES AND CHAIN HOISTS

cable, of sufficient strength to support the permanent 4-eyebar cable, from the continent viaduct floor level up to the top of the continent tower, across the channel to the top of the island tower and down to the island viaduct floor level, this temporary wire cable to be placed approximately 6 ft. higher than the elevation of the permanent eyebar cable. Our intention was to use the wire rope main hangers for the temporary cable, by shipping these hanger ropes to the site in lengths sufficient to stretch across the channel. After serving to erect the two permanent eyebar cables, this rope would be cut up into the correct lengths, socketed and used in the proper position on the bridge for the main hangers.

While Dr. Steinman's very ingenious suggestion of lowering the main cable so as to make the central half of the cable act as the top chord of the stiffening truss resulted in a material saving in steel, it also almost completely eliminated the permanent rope hangers that we had counted upon using for our temporary rope cable. It can be seen from Fig. 1 that the number of linear feet of rope in the hangers in the adopted design was only a small percentage of the amount of rope originally used for the hangers in the conventional design, and therefore some substitute had to be provided.

Erection Cable made up of Flexible Ropes

After giving the matter serious consideration we finally decided upon using a temporary cable made up of flexible ropes of 1-inch diameter, for the reason that these flexible ropes could be used as ordinary hoisting rope in our erection department after serving their purpose during the erection of the eyebar cables.

The permanent hanger ropes were made of 6 strands with a *wire* center, having a modulus of elasticity of from 12,000,000 to 15,000,000. Flexible ropes of 1-inch diameter are made of six strands with a *hemp* center, and have a variable modulus of elasticity, depending upon the length of lay, number of wires to the strand, the method of spinning and also upon the stress in the rope. Our method of erection depended upon supporting the permanent eyebars from the erection ropes by means of hand-operated chain hoists, and then swinging the eyebar cable by releasing or playing out the chain on the hoists, which operation would lower the eyebars until they supported themselves as the permanent cable. During this operation the erection ropes would be relieved of the stress they had while supporting the bars, and would therefore be shortened so that the sag at the center of the span would be materially less and the ropes would move up until they again supported their own weight only.

It was necessary, therefore, to know something about the actual behavior of the erection ropes because our problem was twofold:

1. To make the ropes *short enough* at the start so that when they had elongated under the weight of all the eyebars, the sag at the center would not be so great as to prevent the final bars being placed.
2. To make the ropes *long enough* at the start so that the amount of chain play out on the chain hoists would be enough to permit the eyebar cable to be swung.

If the erection ropes were not short enough at the start, it is conceivable that they would stretch so much under the weight of the eyebars that all of the eyebars could not be placed without great difficulty, and conversely if the erection ropes were made too short. It is also conceivable that when all the chain in the chain hoists had been played out, the eyebars might be still partially supported from the erection ropes.

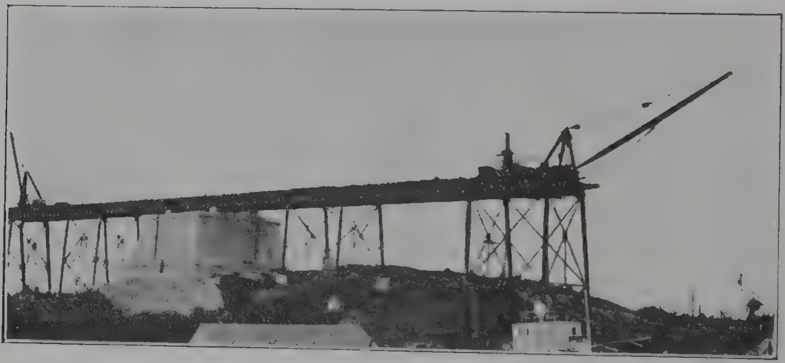


FIG. 10. — TRAVELLER ERECTING ISLAND VIADUCT

The 1-in. diameter hoisting rope has an ultimate strength of about 90,000 lbs. per rope, and with a safety factor of three, a working stress of 30,000 lbs. per rope was permissible. It was found that with this working stress, 24 ropes would be necessary to support the 4 lines of eyebars with their connecting pins. The ends of the 24 ropes were wrapped around rope girders placed just above the floor level of the viaduct, these rope girders in turn being fastened back to the permanent eyebar cable below the top of the viaduct by means of a pair of temporary eyebars. The distance center to center of rope girders was about 1,700 ft.; 50 ft. was added to each end to allow for wrapping, so that each of the 24 ropes was ordered 1,800 ft. long from the Worcester plant of the American Steel and Wire Company.

Tests on Specimens of Erection Cable

To insure as uniform a rope as possible the Wire Company decided to spin 4 ropes of 10,800 ft., each of these 4 ropes to be cut up into 6 lengths of 1,800 ft. each, thus making the 24 ropes. We cut a test piece three ft. long from each end of each 10,800-ft. piece, making 8 test specimens. When the ropes carried their own weight only, the stress per rope was about 3,000 lbs., while under the weight of the entire eyebar cable the stress per rope was about 30,000 lbs. Each of the 8 test pieces was socketed each end and tested in the Emery hydraulic testing machine at Worcester, using an extensometer having a gage length of 10 in. and reading by dials to 0.0001 part of an inch.

An initial load of 200 lbs. was placed on each test piece and called zero stretch with the dials set at zero. Loads were then applied in increments of 200 lbs. up to 3,000 lbs. and a reading of the elongation taken as each increment was applied. Loads were then applied in increments of 1,000 lbs. up to 35,000 lbs. load on the test piece readings being taken at every 1,000 lbs. increment. The load was then released gradually until 3,000 lbs. was reached and the elongation read under this load. Loads were then reapplied a second time in 1,000 lbs. increments until 50,000 lbs. was reached, a reading being taken as each increment was applied. An additional short piece of rope was tested, making nine tests in all.

The reading under the first application of 3,000 lbs. indicated the stretch in the ropes when first placed and supporting themselves. The reading under the first 30,000 lbs. indicated the stretch of the ropes when they supported all the eyebars of the north cable. The reading when the load was released back to 3,000 lbs. represented the stretch in the ropes after the north eyebar cable had been swung and the ropes were again supporting themselves only, and the reading under the second application of the 30,000 lbs. load represented the stretch of the ropes when supporting all the eyebars of the south cable.

The results of these tests were plotted in the form of curves, and these curves were carefully studied. It was found that the average elongation at 30,000 lbs. load was practically the same under both the first and second application of the load, being about 0.1 of an in. in the gage length of 10 in., or approximately 1 per cent. The elongation, under the second application of the 3,000 lbs. load, that is, when the load was released from 35,000 lbs. back to 3,000 lbs., was two or three times as great as the elongation under the first application of the 3,000 lbs. load. This proved that the elongation of the ropes when supporting the second eyebar cable would be practically the same as when supporting the first eyebar cable, and that no extra provision need be made in the chain hoists, as the sag of the ropes under these conditions would be the same. The sag in the ropes, however, after the first cable was swung was considerably more than the sag when the ropes were first put up.

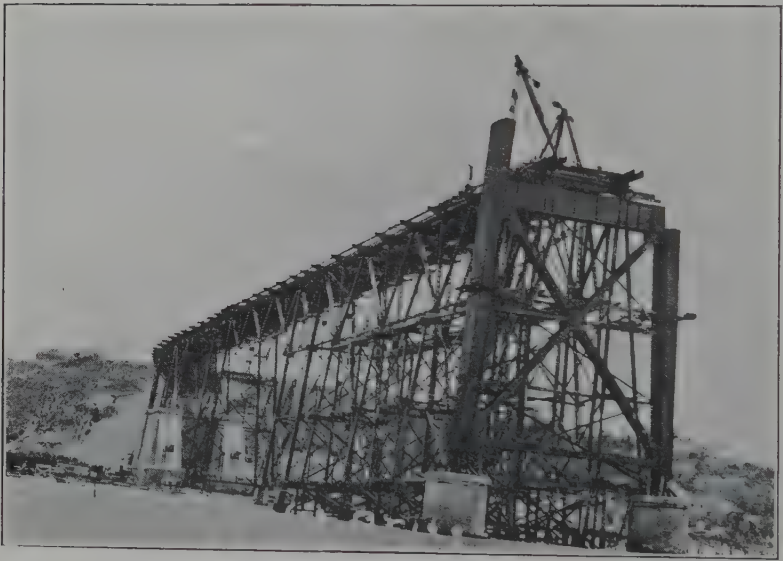


FIG. 11. — TRAVELLER PLACING FOURTH COLUMN SECTION OF CONTINENT TOWER

As a result of these tests we were able to calculate the sag of the erection ropes under the various conditions of loading during the erection of both north and south eyebar cables. As the erection ropes elongated under load, it was necessary to place on rollers the saddles which supported the ropes on top of the towers. The position and movements of these saddles were also calculated from the results of the rope tests. These positions and sags are shown in Fig. 8.*

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* For a complete description of these tests with plots of the curves refer to paper on "Design and Construction of the Florianopolis Eyebar Cable Suspension Bridge" by Steinman and Grove, which will appear in the publication of the American Society of Civil Engineers in 1927.

No difficulty was experienced in the shop fabrication of the material. To insure easy erection each main tower was assembled complete (columns, girders, struts and bracing) in the yards of our Elmira plant, and the field connections reamed and matchmarked.

Full-size tests were made on 12 typical eyebars, all bars passing the elastic limit and ultimate strength requirements, but one bar falling short of the necessary per cent of elongation. An additional full-size eyebar test was made which passed the requirements satisfactorily. The complete physical properties for the 13 test bars are shown in Table I.

TABLE I

PHYSICAL PROPERTIES OF FULL-SIZE TEST EYEBARS

FLORIANOPOLIS BRIDGE

Tests made during May, 1924, and January, 1925

No.	Nominal Section	Length	Elastic Limit (Lb. per Sq. In.)	Ultimate Strength (Lb. per Sq. In.)	Elongation (Per Cent in 18 Feet)	Fracture
1	12″ x 1⅜″	17′–6″	88,000	126,450	8.08*	40% Silky Cup, 60% Silky Sq.
2	12″ x 1⅜″	17′–6″	96,830	137,900	6.43*	Silky Sq.
3	12″ x 1 13/16″	39′–0″	90,100	132,600	6.86	100% Silky Sq.
4	12″ x 1 13/16″	40′–0″	82,810	122,820	7.7	30% Cup, 70% Silky Sq.
5	12″ x 1⅞″	40′–0″	85,710	124,600	7.4	Silky Sq.
6	12″ x 1⅞″	40′–0″	86,740	127,160	6.5	Silky Sq.
7	12″ x 1 15/16″	40′–0″	83,340	120,300	7.2	Silky Sq.
8	12″ x 1 15/16″	40′–0″	78,180	115,970	7.6	Silky Sq.
9	12″ x 2″	40′–0″	83,470	122,500	6.3	Silky Sq.
10	12″ x 2″	40′–0″	79,440	114,660	6.6	Silky Sq.
11	12″ x 2″	40′–0″	81,190	116,820	5.7	Silky Sq.
12	12″ x 2″	40′–0″	82,780	116,720	3.8	Silky Sq.
13	12″ x 2″	40′–0″	79,960	120,350	8.1	Silky Sq.
Average			84,500	123,000	6.8	– –
Minimum requirements . .			75,000	105,000	5.0	– –

* Per cent in 13 feet.

A perusal of this table will show that the elastic limit ranged from 78,180 lbs. per sq. in., 4 per cent higher than the guaranteed minimum, to 96,830 lbs. per sq. in., 29 per cent above the minimum. The ultimate strength ranged from 114,660 lbs. per sq. in., 9 per cent above the minimum requirement, to 137,900 lbs. per sq. in., 31 per cent above the minimum. The average elastic limit was 12 per cent and the average ultimate strength 17 per cent above the minimum requirements. It will thus be seen that a unit working stress of 50,000 lbs. per sq. in. for these eyebars provides an ample margin of safety.

The anchorage steel, consisting of eyebars and girders, was shipped in 1923 and erected by the general contractors, Byington & Sundstrom. The concrete anchorages were then built around this anchorage steel, leaving pockets in the concrete about 3 ft. square surrounding the first panel of bars so as to allow for the eyebar elongation under stress and prevent the breaking of the adhesion of the concrete to the bars.

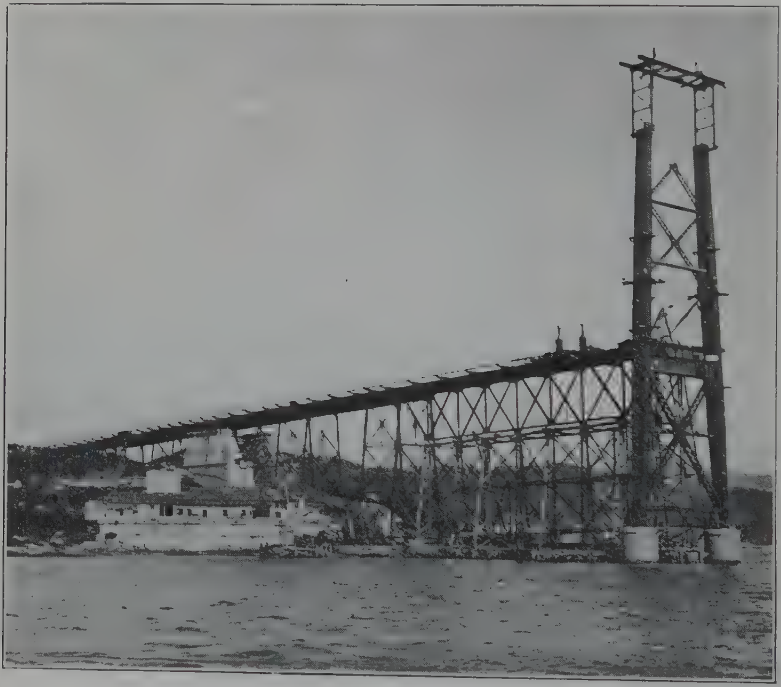

FIG. 12.—CREEPER TRAVELLER USED TO ERECT UPPER SECTIONS OF TOWERS

The balance of the steel was sent from New York in three shipments from June to October, 1924. This steel went by boat the 6,000 miles from New York to the northern end of the Island of Santa Catharina. It was then transferred to lighters and transported down the strait to the material yard which was located on the mainland at the end of the bridge. No difficulties were experienced in transportation, the entire 4,400 tons reaching the bridge site in excellent condition.

METHOD OF ERECTION

The field organization was selected with extreme care and was considered to be the best-equipped erection organization ever sent out by our company on foreign work. The first section of the field forces left New York July 5, 1924, arriving at Florianopolis about August 1, 1924. The entire scheme of erection was a complete departure from methods formerly used in suspension bridge construction, so that every precaution was taken to secure the successful erection of the structure. The organization, consisting of 22 Americans, comprised men who had previous experience in steel erection in Alaska, Mexico, Peru, Uruguay, Colombia, France, Africa and the Australasian Islands. This American nucleus was expanded by the employment of native Brazilian labor, which did most of the handling of material in the yard.

Referring to Fig. 9 the first viaduct tower was erected by a derrick on the ground, and the remaining towers and spans erected by a traveler operating on the viaduct floor level. The 185-ft. spans adjacent to the main towers were erected on falsework, while each truss of the 110-ft. spans was assembled complete on the ground and lifted into position as a unit. The continent and island viaducts were each erected in about three months' time.

The material for the continent and island towers was loaded from the material yard on the lighter and brought to the foot of the respective main piers. The traveler standing at the front end of the 185-ft. span erected the tower shoes, first four sections of columns, with the transverse girders and lower bracing. The upper parts of the fourth column sections were connected back to the U1 top chord point of the 185-ft. span, to prevent the rocker tower from falling over, until the eyebar cables could be erected.

A special creeper traveler was then built around the columns, and this creeper traveler lifted the remaining six sections of tower columns and the upper transverse bracing from the lighters and raised these parts to their proper position in the tower. This creeper traveler climbed the columns as each successive column section was placed.

The continent tower was erected in six weeks, while the island tower erection consumed only five weeks.

Referring to Fig. 9, the eyebars from the continent anchorage X up to viaduct level (point CA6) were erected on falsework attached to the viaduct. The same procedure was followed on the island side from anchorage Y to point IA6.

Rope girders A and E, connected back to points CA7 and IA7,

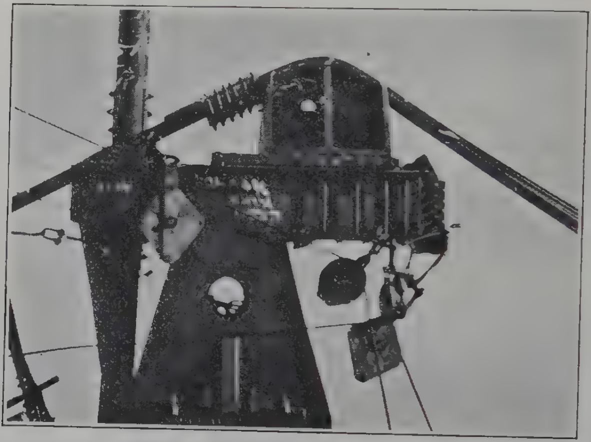

FIG. 13. — ORIGINAL POSITION OF ROPE SADDLE PLACED ON ROLLERS 2′–6″ BACK OF COLUMN CENTRE

Ropes carrying their own weight only

respectively, by temporary eyebars (shown dotted), were placed on the north side of the viaducts. The rope saddles B and D on top of the towers were then placed on rollers which in turn rested on grillages temporarily attached to the tops of the north columns. These saddles were set back from the center of the columns the necessary distances to allow for the stretch of the erection ropes.

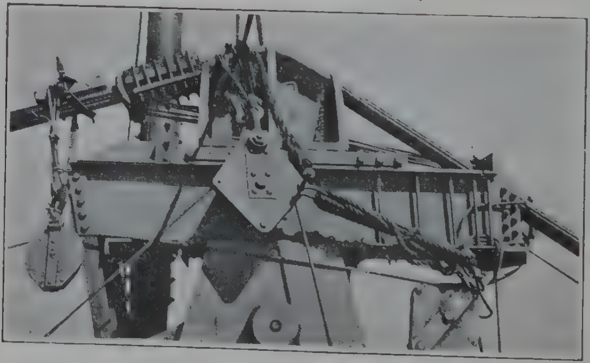

FIG. 14. — ROPE SADDLE OVER CENTRE OF COLUMN WHEN ROPES SUPPORTED ALL EYEBARS

One of the ropes had been marked with wire wrappings at points A B C D and E in accordance with the distances shown on Fig. 8 for the rope under a load of 200 lbs. This pilot rope was pulled over the tops of the towers and stretched until the marks A and E came opposite the pins in rope girders A and E, when the ends of the rope were wrapped one and one-half times around the rope girder and fastened.

The rope was then lifted on the tops of the towers and placed in the saddles in such position that the marks B and D were at the centers of the saddles on top of the continent and island towers, respectively. The mark C midway between B and D was then at the center of the span, and an observation was taken on this mark C to determine the actual sag of the pilot rope under its own weight, and having at that time a stress of 3,000 lbs. The actual sag was between 86 and 87 feet, which closely checked the theoretical calculated sag of 87 feet shown on Fig. 8.

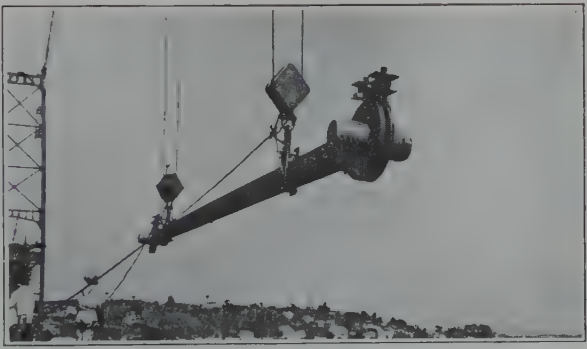

Fig. 15. — Raising Pair of Inner Bars with Pin in Each End

The remaining 23 erection ropes were then placed, making the sag of each the same as the sag of the pilot rope. Three trolleys were then erected on the ropes, one on each backstay, AB and DE, and one on the main span portion of the ropes, BD. The main span trolley moved across the ropes from island tower to continent tower clamping the 24 ropes together to form a single cable and placing the 26 chain hoists at the panel points shown on Fig. 9. Each backstay trolley was meanwhile clamping the backstay portion of the 24 ropes together and placing the rod attachments for the eyebars.

All the eyebars for the main span, about 16 bars at a time, were placed on the lighter, floated to a position under the erection ropes and lifted into position by the main span trolley. To avoid the necessity of driving the pins in midair, the two inner bars of alternate panels were assembled in the yard and pins placed in each end. In this way the bars were all slipped over the pins, reversing the usual procedure, and eliminating platforms at each panel point which would be necessary were the pins driven through the eyebar pin holes. The pins at the top of the towers were driven through the eyebars, but there we had the top transverse tower strut to use as a working platform.

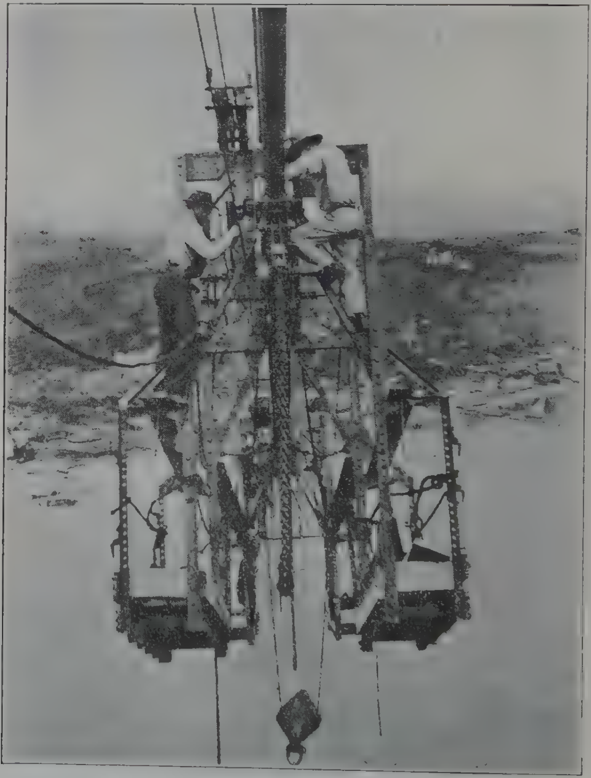

FIG. 16. — MAIN SPAN TROLLEY

The first eyebars lifted into position were near the center of the main span, this portion of the ropes being loaded to bring the rope saddles over the centers of the towers as soon as possible. When 25 per cent of the main span eyebars had been lifted into position and supported from the chain hoists, it was safe to proceed with the erection of the backstay eyebars because by that time the saddles had moved on the rollers nearer to the center of the columns. It was not possible for the saddles to roll off the columns forward because the difference in length between the arc and the chord of the backstay ropes was only about 2 to 3 in. On the main span portion of the ropes, however, the arc was about 20 ft. greater than the chord, so that it was possible for the saddles to roll off the towers backwards, if the backstay part of the ropes had been loaded with eyebars without any counterbalancing eyebars on the main span portion of the ropes.

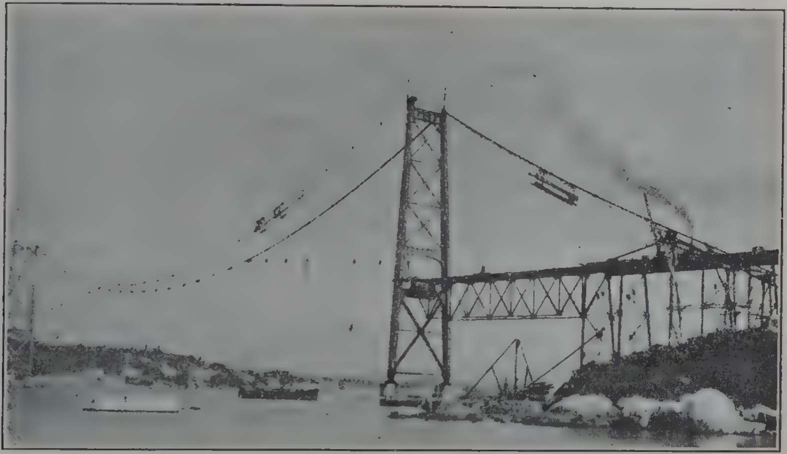

Fig. 17. — Erecting Eyebars for South Cable
North cable already swung

It took about ten days to erect the ropes, trolleys, clamps and chain hoists, and two weeks to erect the 156 eyebars in the north cable. The last eyebar was placed on Saturday, March 7, 1925, and the actual sag of the ropes when carrying all the eyebars measured 113 feet as against the theoretical calculated sag of 115 feet shown on Fig. 8. This difference is due to not placing the gusset plates and hangers as was originally intended when the calculations were made. The saddles at points B and D had moved over the centers of the columns.

After lunch on March 7, 1925, men were stationed on the eyebars at the chain hoists, the eyebars being entirely supported by the ropes and at a uniform distance of 6 ft. below the ropes. At 2.30 P.M. the signal was given for all men to slacken off the chain hoists, and in a few minutes the 6-ft. gap began to widen. By 3.15 P.M. all 26 chain hoists were slack, indicating that the north eyebar cable was swinging under its own weight, and that the ropes were free of the bars. An observation showed that the actual sag of the ropes was between 99 ft. and 100 ft. as against a theoretical calculated sag of 99 ft. shown on Fig. 8. The entire operation of swinging took only forty-five minutes and was a complete success in every respect.

The main span trolley then removed the chain hoists and clamps, and at the same time placed the rope hangers and structural steel hangers. Simultaneously the backstay trolleys removed the clamps and rod connections on those portions of the ropes. The next operation was to transfer the ropes from the north to the south side of the bridge in order to erect the south eyebar cable. The ropes were first lifted out of the saddles and laid on the top transverse tower struts one at a time, the ends disconnected from the rope girders and temporarily fastened to the viaduct floor. Referring to Fig. 9 the rope girders and temporary eyebars at A and E were transferred to corresponding positions on the south side of the bridge, and the saddles, rollers and grillages on the tops of the towers at B and D were transferred to the tops of the south columns. The 24 ropes were then refastened to the rope girders and

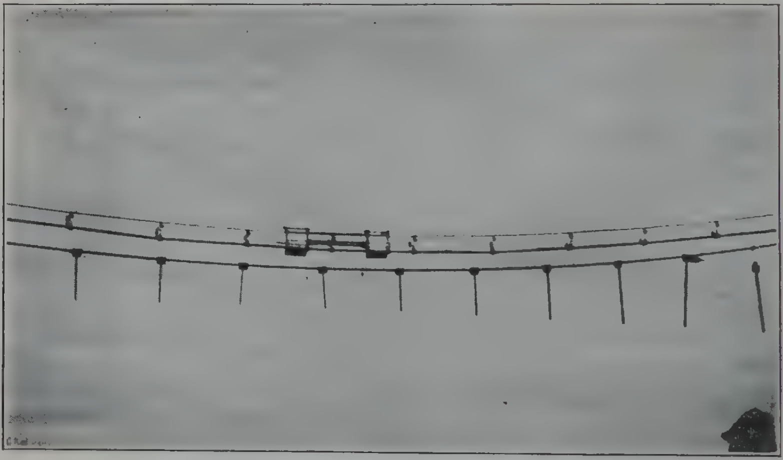

FIG. 18. — ERECTION OF SOUTH EYEBAR CABLE

South eyebar cable still supported from erection ropes. Note man standing at each panel point on eyebars ready to start releasing chain on hand operated chain hoist. (North eyebar cable already swung) replaced in the saddles in the same manner as before. The three trolleys were re-erected on the ropes and the rope clamps, rods and chain hoists placed as previously for the north cable erection.

The erection of the south eyebar cable was proceeded with in the same manner as for the north cable, and by April 2, 1925, the entire 156 eyebars were supported from the ropes. The actual sag of the ropes was again 113 ft., the same as for the north cable, as against the theoretical calculated sag of 115 ft. shown on Fig. 8, but again the omission of gusset plates and hangers accounted for the difference. This same distance of 113 ft. in both cases checked up the original tests of the rope specimens which showed that the elongation of the ropes under the second application of the 30,000 lbs. load was practically the same as the elongation under the first application of the 30,000 lbs. load. As before, men were placed at the chain hoists, and at 3.30 P.M. the signal was given to slacken off. By 4 P.M. all chain hoists were slack and the south eyebar cable was swinging free. The operation this time took only thirty minutes. The trolleys then removed clamps and chain hoists, the main span trolley at the same time placing the rope hangers and structural steel hangers. The three trolleys were then taken down.

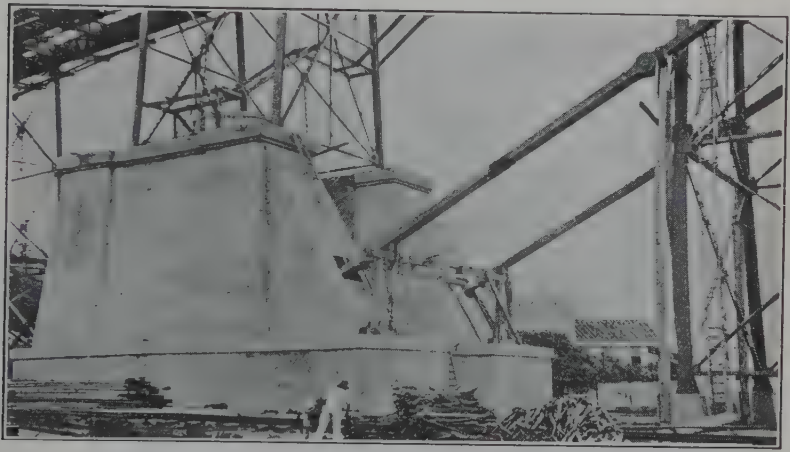

FIG. 19. — CONTINENT ANCHORAGE WITH BACKSTAY EYEBARS

The operation of placing the 24 ropes, trolleys, clamps and chain hoists for the north cable took twelve days. The erection of the north eyebar cable (156 eyebars) took two weeks. Placing north hangers, removing clamps, chain hoists and trolleys, transferring ropes, rope girders, saddles and grillages, re-erecting trolleys and replacing clamps and chain hoists took about two weeks. The erection of the south eyebar cable (156 eyebars) took one week, just one-half of the time consumed during the erection of the north cable.

Referring to Fig. 9, it will be seen that no portion of the approaches are supported from the cables. The sag of the main cables under full dead load was to be 120 ft. When the cables were swung under their own weight only, the sag of the cables was 116 ft., and at that time each tower was tilted back about 1 ft. The towers were therefore erected in this position when the tops of the fourth column sections were locked into the 185-ft. span by temporary struts, and after the cables were swung, these temporary struts were removed. As the main span was erected, the sag of the cables would increase from 116 ft. to 120 ft., and the towers would revolve about the base until they assumed a vertical position under full dead load.

Before the stiffening truss was connected up the entire weight of the bridge was supported by the eyebar cables which were flexible from tower to tower. The bridge was designed on the assumption that all the dead load was to be carried by the cables, the stiffening truss being designed for live load and temperature only, and not to carry any dead load.

The cable, being flexible during the erection of the bridge, would assume equilibrium polygons of constantly changing shapes, depending upon the loads applied at the 26 panel points. Adjustment was provided where the rope hangers joined the top chords at panel points 2 to 14, inclusive, but no adjustment was provided in the central portion of the span between panel points 16. When all the dead load was on the bridge the cable would hang in some equilibrium polygon, and while the shape of this polygon could be determined on the basis of known loads, it was not considered safe to depend entirely upon calculations, as the distance center to center of towers could not be determined accurately, owing to conditions at the site which did not admit of accurate triangulation. We therefore decided to ship the 22 diagonals between points 16 with holes in each end, but to send the bottom chord gusset plates with holes blank, for the connection of these 22 diagonals, and after the dead load was on the span to drill the holes in the bottom gusset plates for the connection of these diagonals, using the diagonals themselves as templets. With the entire dead load on the cables the sag would be 120 ft. at normal temperature, and by drilling at that time, the diagonals could be connected under zero stress.

As the eyebar cable was a constantly changing equilibrium polygon during the erection of the main span, there was considerable doubt in our minds as to the practicability of erecting the main suspension span by the ordinary traveler method of operating on the main deck with travelers advancing simultaneously from each tower, because the extra weight at panel points near the tower would distort the cable so far from its final curve as to prevent the placing of some of the truss members.

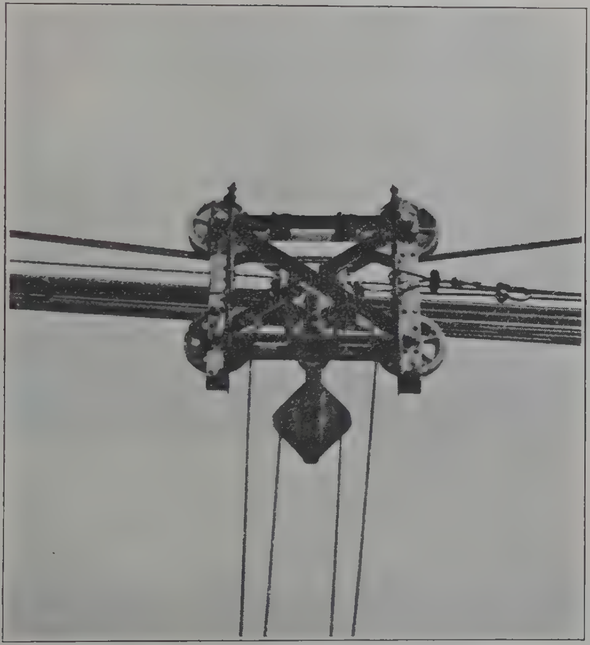

Fig. 20. — Small Trolley Used to Erect 1,000 Main Span Members

Equilibrium polygons were made showing conditions when simultaneous erection from each end had proceeded to the 4th, 8th, 12th, 16th, 20th and 24th panel points.

Like the eyebars, all the main span material was brought out in a lighter under the cables and lifted into proper position on the bridge. The first two panels at each end of the main span truss were erected by travelers standing on the viaducts at the main towers. When the traveler moved out under point 2 (see Fig. 9) to place panels 3 and 4, point 2 of the cable moved down so much that top chord 2–4 of the stiffening truss could not be placed, thus bearing out the conclusion reached by a study of the equilibrium polygons. This contingency had, however, been provided against, and the traveler was moved back on the viaduct and the balance of the main span was erected by the overhead trolley method.

After the south eyebar cable had been swung, the 24 erection ropes had been temporarily left over the south columns. Two of these ropes were now lifted from the saddles at B and D and placed on wheels resting on the top transverse tower strut adjacent to the eyebar cable shoe on top of the column. The ends of these two ropes were unfastened from the rope girders A and E and wound around the pins at the center of these rope girders, and a small trolley was placed on these two ropes. Another pair of ropes was lifted out of the saddles and placed on wheels on top of the transverse tower strut near the north eyebar cable, and another small trolley was placed on this pair of ropes.

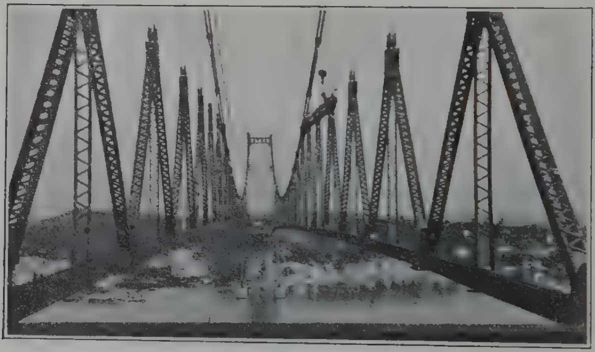

FIG. 21. — ERECTING TOP CHORDS OF NORTH TRUSS

View looking through the span. Bottom chords and diagonals of both trusses already in place

These two trolleys, weighing only 5,000 lbs. each, and controlled by engines located on the viaducts at the main towers, erected the entire 1,000 members of the 54 panel main span, — bottom chords, diagonals, top chords, floorbeams, stringers, bottom laterals, sidewalk brackets, top laterals and top struts, with the exception of the two end panels at each end of the bridge. The bottom chords were first connected to the hangers, the diagonals were then placed and next the top chords. In this way the load on the eyebar cables was kept as uniform as possible. When all truss members were placed, the floorbeams and bottom laterals were erected, followed by the stringers and top bracing. The main span erection consumed forty-five working days.

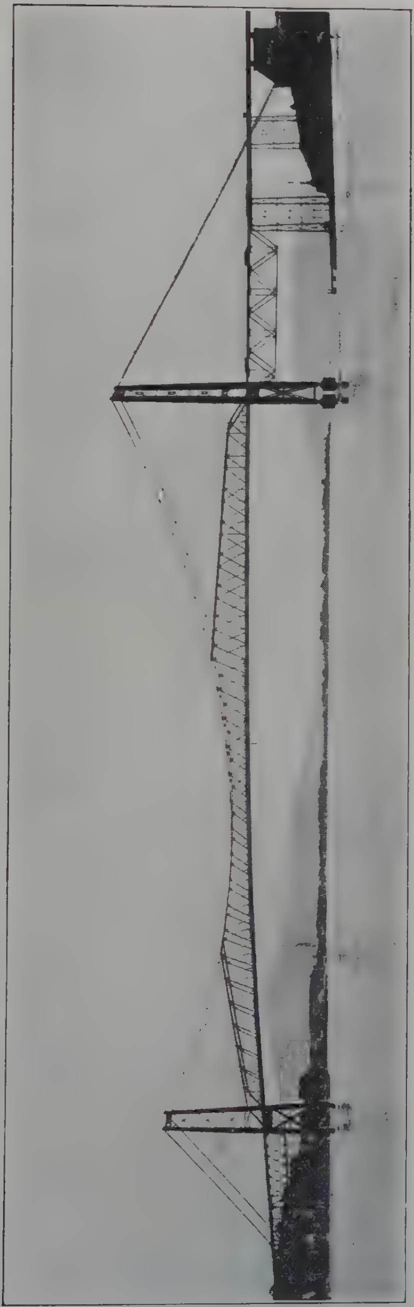

FIG. 22. — VIEW OF COMPLETED FLORIANOPOLIS BRIDGE

When all steel work had been placed, only about two-thirds of the entire dead load was on the cables, and the sag of the cables therefore had not gone down to the proper distance of 120 ft. There was a delay in the delivery of the floor lumber, so in order to drill the holes in the bottom chord gusset plates to connect the 22 diagonals between points 16, it became necessary to provide some temporary load to bring the sag down to the proper amount. There was plenty of sand along the beach on the continent shore, so some of this sand was placed on the steel floor to provide for a load equivalent to 1,400 lbs. per linear foot. Under this temporary load the sag was brought to 120 ft., and the diagonals at each end from point 0 to point 16 could be connected with practically no drifting. Using the holes in the diagonals between points 16 as templets, the 1,728 holes in the bottom chord gusset plates were drilled and these diagonals then connected. All the gusset plates were ½ in. thick, and one of the erectors drilled 240 holes himself in one day of eight working hours.

The 50,000 field units in the main span were driven, the temporary sand load removed, and the entire structure given the field coats of paint.

The last contingent of our erection forces arrived in the United States on August 31, 1925, so that fourteen months elapsed from the time the first section of the field organization left this country until the last section returned; the actual time of work at the site being just about one year to erect the 4,400 tons of steel.

The wood floor was placed by the general contractors, Byington & Sundstrom, and completed during March, 1926. The official opening ceremonies took place on May 3, 1926.

Conclusion

The Florianopolis Bridge is novel in four respects:

1. It is the first suspension bridge where a portion of the cable is integral with the top chord of the stiffening truss.
2. It is the first American suspension bridge making use of rocker towers.
3. It is the first bridge of any kind utilizing heat-treated eyebars of the 75,000 lbs. per sq. in. elastic limit grade.
4. In the construction of this bridge the overhead method was used for the erection of the entire cables and main span, eliminating ordinary falsework and working platforms. Not one piece of falsework was placed in the channel from pier to pier.

In addition to the above special features it has the distinction of being the longest eyebar cable suspension bridge in the world.

It is a practical type of suspension bridge because it has been completed, and no unusual difficulties were encountered in its construction.

It is a safe bridge to construct because the entire operations covering practically a year were accomplished in a foreign country 6,000 miles from our base without any fatality.

It is an economic type of suspension bridge because in competition with cables of wire and wire rope, the estimated cost of heat-treated eyebar cables was lower than the estimated cost of wire cables or wire rope cables, and the actual cost of the bridge was inside of the estimate.

The eyebar cable suspension bridge is therefore a practical, a safe, and an economic type of suspension bridge.

A particularly gratifying feature from an engineering standpoint was the manner in which the actual movements of the erection ropes checked up the calculations. But a more gratifying feature to me was the fact that this new type of bridge with all its novel features was successfully erected 6,000 miles from home without the loss of a single human life.

OF GENERAL INTEREST

NEW WATER SUPPLY PROJECT UNDER WAY

Metropolitan District Water Supply Commission Making Surveys for Ware River Tunnel and Swift River Reservoir — Frank E. Winsor is Chief Engineer

After several years of investigation, legislative action was finally secured last spring which resulted in the appointment of the Metropolitan District Water Supply Commission with the following personnel: Davis B. Keniston, Chairman, Metropolitan District Commission; Charles M. Davenport and Joseph H. Soliday.

The Commission was authorized to design and build a tunnel from the Wachusett Reservoir near Oakdale to the Ware River at Coldbrook, Mass.; and to purchase land for the Swift River Reservoir in Enfield, Greenwich, Prescott, Dana, New Salem and vicinity. It was also directed to report to the Legislature on the first Wednesday in January, 1927, recommending legislation for carrying out the Swift River Project. An appropriation of $15,000,000 was provided for construction of the Ware River Tunnel and purchase of lands for the Swift River Reservoir.

Mr. Frank E. Winsor, Chief Engineer for the New Providence (R. I.) Water Supply, was appointed as Chief Engineer for the Commission. Mr. Winsor has had a broad experience on water supply matters. After a number of years on the Catskill Aqueduct of the New York water supply, Mr. Winsor became Chief Engineer of the New Providence Water Supply. The construction work in connection with the Providence project was nearly completed at the time of the appointment of the Commission for the Metropolitan District Water Supply, so that it was possible for Mr. Winsor to accept the appointment as Chief Engineer for this new and important development in Massachusetts.

Karl R. Kennison has been appointed Designing Engineer; and N. Le Roy Hammond and Harold W. Horne have been appointed Assistant Division Engineers.

Preliminary surveys for the Ware River tunnel are now being made under the direction of Mr. Horne. This tunnel, about 13 miles long, will be well underground with shafts varying from 200 ft. to 600 ft. deep. It will probably be of horseshoe section equivalent to 12′ 9″ circle. A contract has already been awarded for borings in connection with this tunnel.

Mr. Hammond is engaged on surveys for the Swift River Reservoir, and a contract has been let for borings for the Swift River dams.

Eventually the Ware River tunnel will

be extended west to the Swift River, making the total length of tunnel about 25 miles. The Commission is actively engaged in studies for the Swift River development, as it was directed to report to the incoming Legislature with recommendations for legislation for carrying out that project.

B. S. C. E. Trip to Alden Hydraulic Laboratory

An interested group of members of the Boston Society of Civil Engineers visited the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute on Thursday, November 11. The trip, made by automobile from Boston, was scheduled for Tuesday, November 9, but owing to unfavorable weather was postponed to Thursday.

This laboratory, made possible through the generosity of the late George I. Alden, former head of the Mechanical Engineering Department of Worcester Polytechnic Institute, is under the direction of Charles M. Allen, Professor of Hydraulic Engineering.

A description of the laboratory, which is located in Holden about 5 miles from the city of Worcester, was given in the Journal of the Boston Society of Civil Engineers in May, 1926.

After an inspection of the plant and equipment, including an observation of the regular work under way in the laboratory, refreshments were served to the visitors through the hospitality of Professor Allen, the President of the Boston Society of Civil Engineers, who has been in active charge of the laboratory from its early days to its present stage of development.

New Members

There is probably some one in your office who would be interested in joining the Society if he were familiar with its activities and realized what it had to offer. Why not bring him to a meeting? Guests are always welcome.

A line to the Secretary will bring an "application" blank.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

October 20, 1926. — A joint meeting of the Boston Society of Civil Engineers; M. I. T. Student Chapter, American Society of Civil Engineers; Harvard Engineering Society; Tufts Civil Engineering Society; and Northeastern University Section, B. S. C. E., was held today in Chipman Hall, Tremont Temple, and was called to order at 7.30 P.M. by Professor C. M. Allen, President of the Boston Society of Civil Engineers. About 300 members and guests attended.

In opening the meeting President Allen stated the pleasure it gave the Boston Society of Civil Engineers to take part in a joint meeting with the Student Engineering Societies of Technology, Harvard, Tufts and Northeastern University. He extended a cordial invitation to the members of these societies to attend any meetings of the Boston Society of Civil Engineers, and to make use of the rooms of the Affiliated Technical Societies of Boston, at 715 Tremont Temple, Boston.

Professor Allen then introduced Dr. D. B. Steinman, Consulting Engineer of New York City, who gave an illustrated talk on the design of the Florianopolis Bridge which connects the mainland of Brazil with Florianopolis, the island capital of the State of Santa Catharina. After Dr. Steinman's talk, Mr. William G. Grove, of the American Bridge Company, was introduced and he gave an illustrated talk on the construction features of the Florianopolis Bridge, together with a travel talk of a trip to South America, which he had made at the time of the construction of the bridge.

At the close of the meeting a vote of thanks was given to Dr. Steinman and to Mr. Grove for their courtesy in coming to Boston and presenting these interesting papers.

A buffet supper and smoker was held in advance of the meeting. Following the supper the groups from the different student societies sang a number of their college songs.

Meeting adjourned at 10.45 P.M.

J. B. BABCOCK, *Secretary.*

APPLICATIONS FOR MEMBERSHIP

[Nov. 15, 1926]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

For Admission

CONSTANTINE, BASIL GEORGE, Arlington Hts., Mass. (Age 23, b. Sparta, Greece.) Graduate of Massachusetts Institute of Technology. From June to September, 1925, structural draftsman, Board of Transportation, Division of Designs, New York, N. Y.; June, 1926, to date, structural draftsman on designs and repairs to overhead and railroad bridges, Boston & Maine Railroad. Refers to H. K. Barrows, B. W. Guppy, Pusey Jones, C. M. Spofford.

DAVIS, CHARLES EUGENE, Arlington Heights, Mass. (Age 42, b. Westbrook, Me.) Graduate of University of Maine with degree of B.S. in civil engineering in 1907, and in 1912 granted degree of C.E., having specialized in computation and design of track work for street railways. From 1907–09 was with the Bethlehem Steel Company as draftsman in track work; from November, 1909 to May, 1926, employed by Barbour, Stockwell Company as engineer in track work; and since May, 1926, with the Boston Elevated in track work engineering. Refers to F. C. Bell, H. S. Boardman, B. W. Ellis, H. C. Hartwell, H. M. Steward.

EARLE, RALPH, Worcester, Mass. (Age 52, b. Worcester, Mass.) Graduate of the United States Naval Academy. After graduating from Annapolis held various commissions, — in December, 1916, was Chief of Bureau of Ordnance with rank of Rear Admiral. In May, 1920, after all his war contracts had been adjusted, he left the post of Chief of Ordnance for sea duty in command of the "Connecticut", taking rank as Captain. In 1921 he was made Chief of Staff of Control Force, with the U. S. S. "Florida" as flagship; on May 31, 1922, he took command of the Naval Torpedo station at Newport, R. I. In the spring of 1925 he received leave of absence in order to assume the Presidency of Worcester Polytechnic Institute, his official retirement from active Naval service occurring on August 25, 1925. Refers to C. M. Allen, H. P. Eddy, A. W. French, I. N. Hollis, E. A. Taylor, F. O. Whitney.

Franz, Herbert J., Dorchester, Mass. (Age 38, b. Boston, Mass.) From 1906 to 1912 employed by Lewis H. Bacon as architectural draftsman; then was employed by Blackall, Clapp & Whittemore as draftsman until November, 1925, during which time he took a course in structural design at the Boston Architectural Club under Prof. Charles Killam of Harvard; from November, 1925, to date has been designer with Monks & Johnson. Refers to Mark Linenthal, W. F. Pike, B. A. Rich, E. A. Varney.

Gilmore, Ross Angus, Quincy, Mass. (Age 21, b. Quincy, Mass.) A student at Northeastern University, class of 1927. During his Sophomore and Junior years co-operated with Ernest W. Branch, and is working for the Mass. Dept. of Public Works, Highway Division. Refers to H. B. Alvord, E. W. Branch, I. H. Call, R. C. Chapin, J. W. Ingalls, W. E. Nightingale.

Kingman, Edward Dyer, Brookline, Mass. (Age 41, b. Holliston, Me.) Educated in public schools of Framingham, and graduate of Yale University; 1907–11, in field and office work; 1911–13, instructor in civil engineering, Michigan State College; 1913–15, assistant professor, civil engineering, University of Wisconsin; 1916, on valuation work; 1917 assistant professor, civil engineering, University of Maine; also editorial assistant; 1918 to date, instructor at Wentworth Institute, department of applied mechanics and materials of construction, also editorial assistant for McGraw Hill Publishing Company of New York. Refers to J. B. Babcock, C. W. Banks, H. S. Knowlton, E. A. Varney.

Lewis, Edgar V., Boston, Mass. (Age 21, b. Middleborough, Mass.) Is a student at Northeastern University. Refers to G. A. Haskins, J. W. Ingalls, W. E. Nightingale, A. M. Pillsbury.

Lofgren, Rudolph A., Quincy, Mass. (Age 21, b. Quincy, Mass.) A student at Northeastern University. During work periods has been employed by Metropolitan District Commission, Park Division, since 1925, both as rodman and transitman. Refers to H. B. Alvord, I. H. Call, R. C. Chapin, J. W. Ingalls, W. E. Nightingale, Stanley Szlanda.

Marston, Francis John, Dorchester, Mass. (Age 21, b. Dorchester, Mass.) Graduate of Dorchester High School. In October, 1924, worked as rodman for Ernest W. Branch. Entered Northeastern University in February, 1925. During work periods has been employed by E. W. Branch. Refers to C. O. Baird, E. W. Branch, G. A. Haskins, J. W. Ingalls.

McGrath, Russell P., Brookline, Mass. (Age 22, b. Albany, N. Y.) A student at Northeastern University. During work periods has been employed by the town of Windsor, Conn., and with the Turner Engineering Company of Windsor. Refers to H. B. Alvord, C. O. Baird, J. W. Ingalls, W. E. Nightingale.

Norcross, Vernon Norman, East Bridgewater, Mass. (Age 19, b. Pembroke, Mass.) A student at Northeastern University. During work periods has been employed by Fred R. Charnock, City Engineer, Medford, and Hayward & Hayward, C.E. of Brockton. Refers to C. S. Ell, J. W. Ingalls, G. A. Haskins, W. E. Nightingale, A. M. Pillsbury.

Rucker, Allen E., Boston, Mass. (Age 25, b. Jersey City, N. J.) Employed in the Engineering Dept., Standard Oil Company of New York. Refers to T. J. Conroy, L. C. Fuller, H. C. Harris, W. L. Thompson, E. A. Varney.

Thompson, George Donald, Dorchester, Mass. (Age 22, b. Dorchester, Mass.) A student at Northeastern University. Worked with C. H. Gannatt from August, 1925, to August, 1926; at present with C. Frederick Joy as transitman on sewers, Milton, Mass. Refers to W. J. Alcott, Jr., H. B. Alvord, C. S. Ell, J. W. Ingalls.

Wistreich, Arthur I., Dorchester, Mass. (Age 19, b. Dorchester, Mass.) A student at Northeastern University. During the summer of 1925 was rodman for C. H. Gannatt; then transferred to town engineer's office of Norwood, where he was transitman; and from the spring of 1926 was transitman and draftsman with Schein & Levine, Boston. Refers to C. O. Baird, C. S. Ell, J. W. Ingalls, W. E. Nightingale.

For Transfer from Grade of Junior

JUNIOR, FRANCIS EDMUND, Tampa, Fla. (Age 25, b. Taunton, Mass.) Graduate of Northeastern University in 1922. During work periods while in college was employed by Whitman & Howard as rodman and transitman; with Paul H. Delano as transitman; with the Boston & Albany Railroad Department of Maintenance of Way; November, 1922, to February, 1923, with Alonzo B. Reed as chief of party and mechanical draftsman; February, 1923, to April with Colonial Filling Station as architectural draftsman; April to August, 1923, with Arlmont Country Club as engineer on golf course construction; August to December, 1923, with Wayne E. Stiles as timekeeper and assistant on golf course construction; December, 1923, to April, 1926, with Worth C. Peacock, town planner and consulting engineer, at Tampashores, Fla., as assistant engineer on development and construction; April to June, 1926, with Florida Interurban Rapid Transit Railway Company as chief of party; June, 1926, to date with same company as acting chief engineer. Refers to H. B. Alvord, C. S. Ell, F. L. Flood, Channing Howard, W. E. Nightingale, A. B. Reed.

TSAGARIS, DEAN PETER, Lowell, Mass. (Age 24, b. Greece.) Graduate of Tufts College in 1924 with degree B.S. in civil engineering. Previous to receiving degree was employed by Aberthaw Company. From June to September, 1924, did private work on surveys and house planning; from September, 1924, to September, 1925, was structural steel designer with McLaughlin & Burr, and for three months of this time was engineer for the firm; from September to November, 1925, was structural designer with Monks & Johnson; from November, 1925, to March, 1926, was with the Boston & Maine Railroad as structural draftsman; March to August, 1926, with E. N. Pike; with Jackson & Moreland during August; at present with Stone & Webster, Inc., as structural designer of buildings, power plants and transmission lines. Refers to Robinson Abbott, E. D. Mortenson, W. F. Pike, F. N. Weaver, E. H. Wright.

NEW MEMBERS

HENRY B. CAMPBELL, 408 Atlantic Ave., Boston, Mass.

MORGAN L. WOODRUFF, 862 Park Square Bldg., Boston, Mass.

BOOK REVIEW

*"Reinforced Concrete Design," by Hale Sutherland, Associate Professor of Structural Engineering, Mass. Inst. of Technology, and Walter W. Clifford, Consulting Engineer, Boston. 1st Edition, 1926. John Wiley & Sons, Inc., New York, 412 pages.**

This work is designed primarily as a textbook for classroom work and not as a handbook for designers.

After a short introduction and historical note the authors give brief descriptions of materials, proportioning, mixing and placing, and the physical properties of concrete and steel. Forms are next considered. Then follows the main feature of the work — design.

In the analysis of beams and compression members the authors have resorted to the transformed section. This method facilitates the presentation of the subject and obviates too much reliance on formulas. The authors, however, recognize the impracticability of the common use of this method in practice, and have presented a chapter on Formulas, Diagrams and Tables intended to familiarize the student with the labor-saving features used in office practice.

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* Reviewed by John J. Harty, Consulting Engineer, Boston; Lecturer in Structural Engineering Mass. Inst. of Technology, Cambridge, Mass.

Considerable attention has been rightly given to the exact determination of stresses in Continuous Beams and Rigid Frames. The continuous beam, least work and slope deflection theories are developed.

Various methods are given for proportioning and analyzing arches.

Throughout the work the authors have arranged computations on parallel pages with the text giving set forms for these computations. Inasmuch as the forms for computations are usually different in different offices and for different kinds of problems and building codes, it would seem unwise, in a textbook, to stress the use of any particular set of forms.

The work closes with chapters on Plans and Details and on Economy. There are appended, Rankin's Theory of Earth Pressure, Theory of Hingeless Arch, Tables and Diagrams for Design, and excerpts from the Joint Committee Specification.

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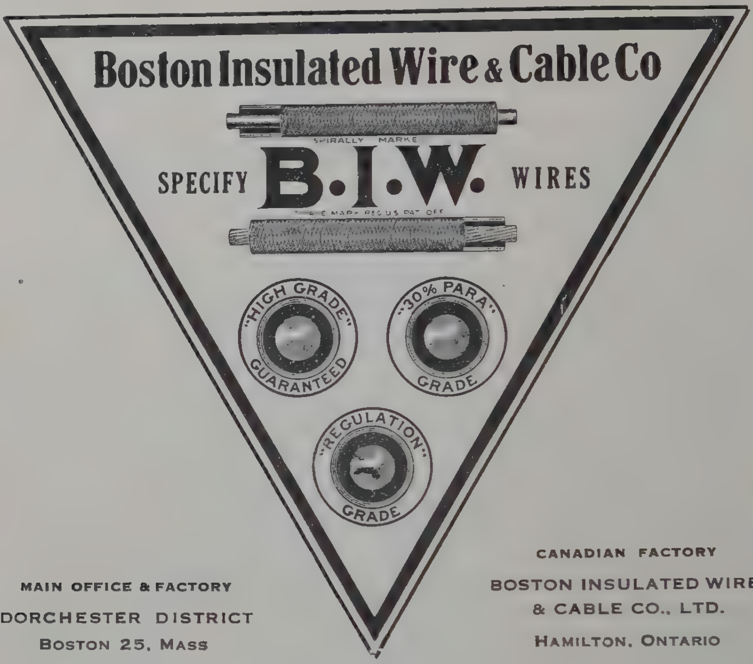

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